CONCRETE
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CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

FEBRUARY, 1952. **Vol. XLVII, No. 2**

FORTY-SEVENTH YEAR OF PUBLICATION

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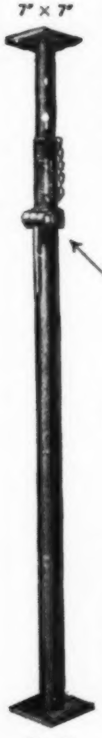

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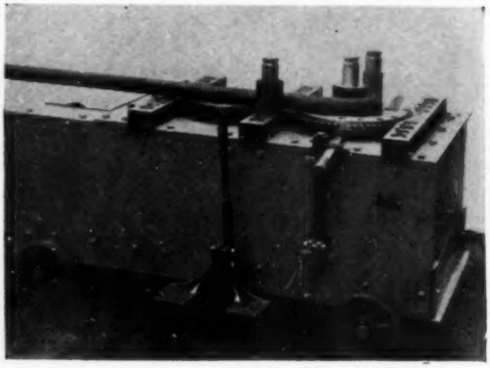

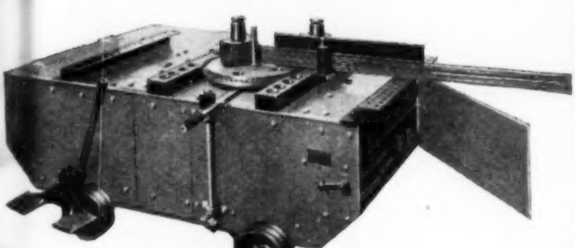

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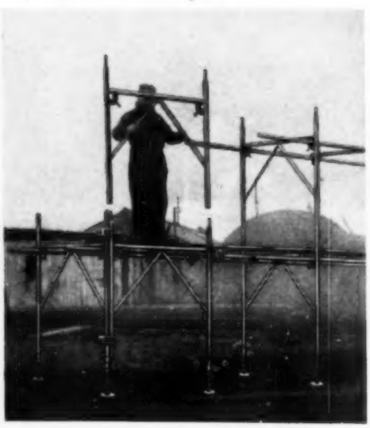

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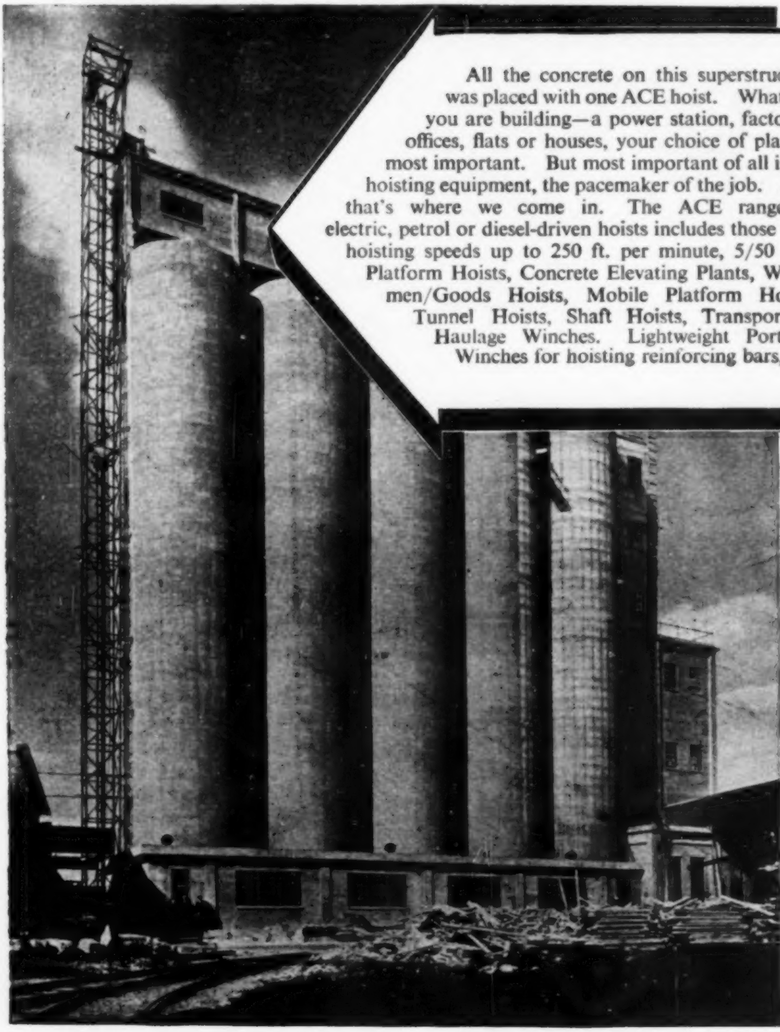

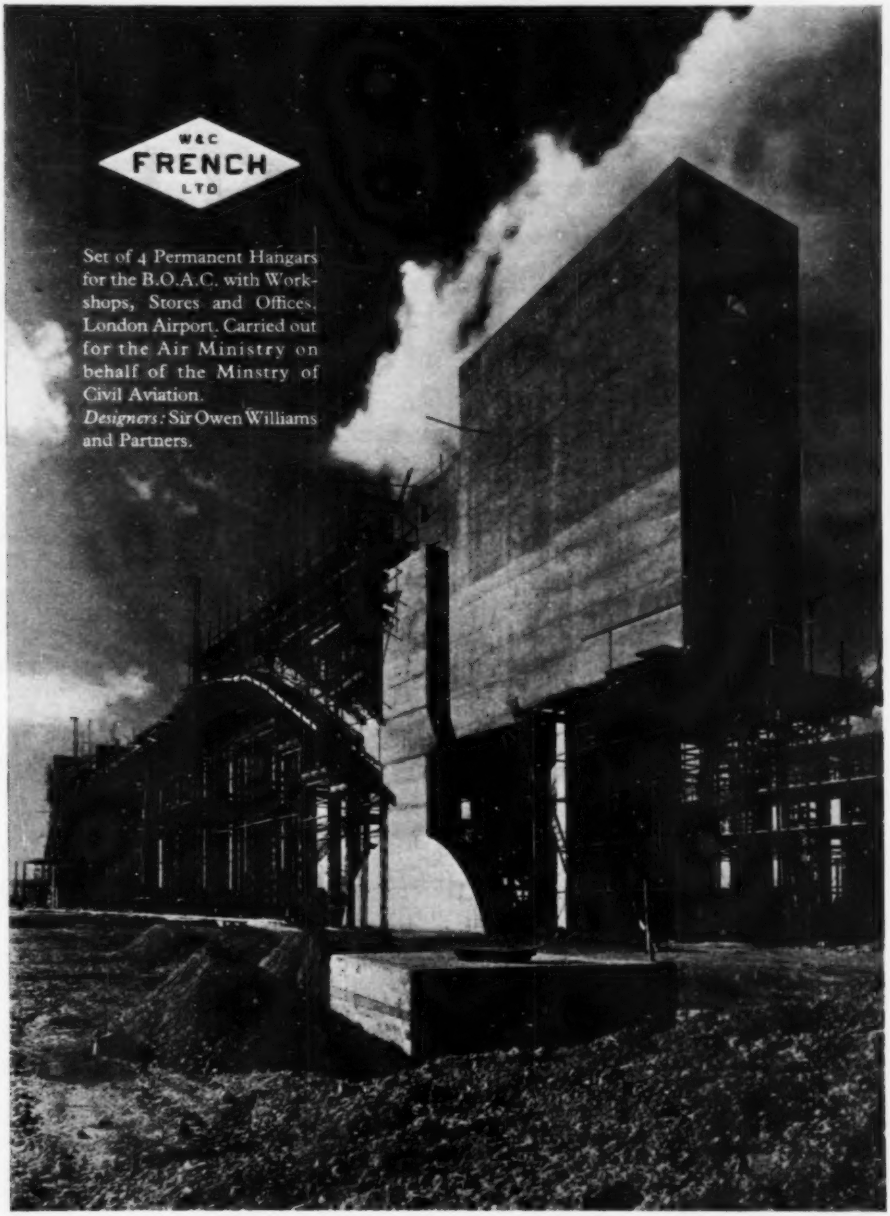
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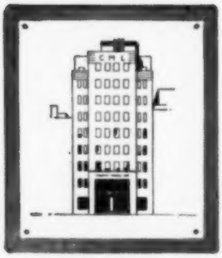

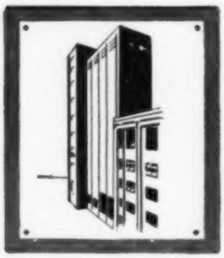

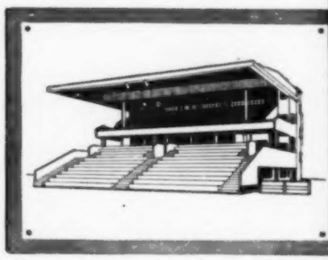

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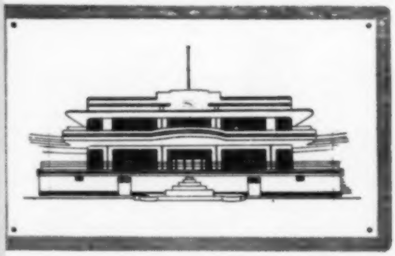

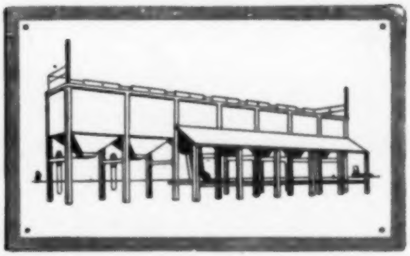

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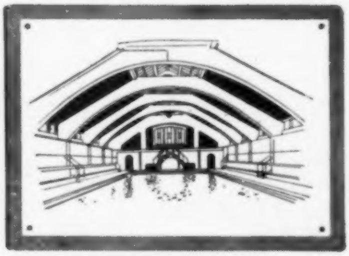

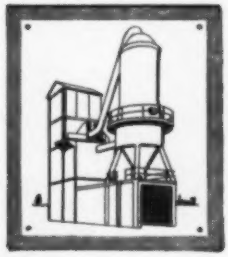

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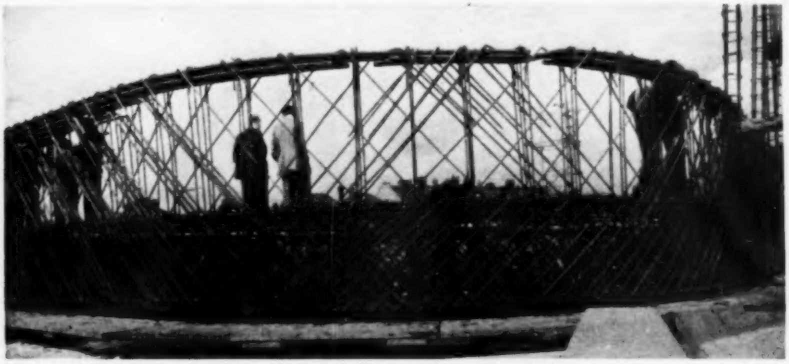
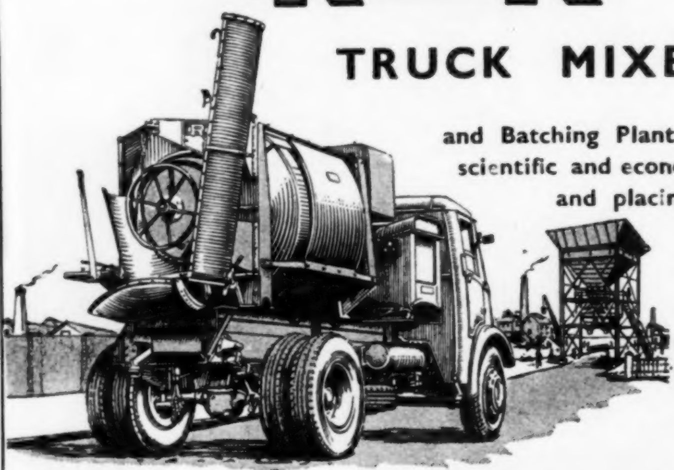
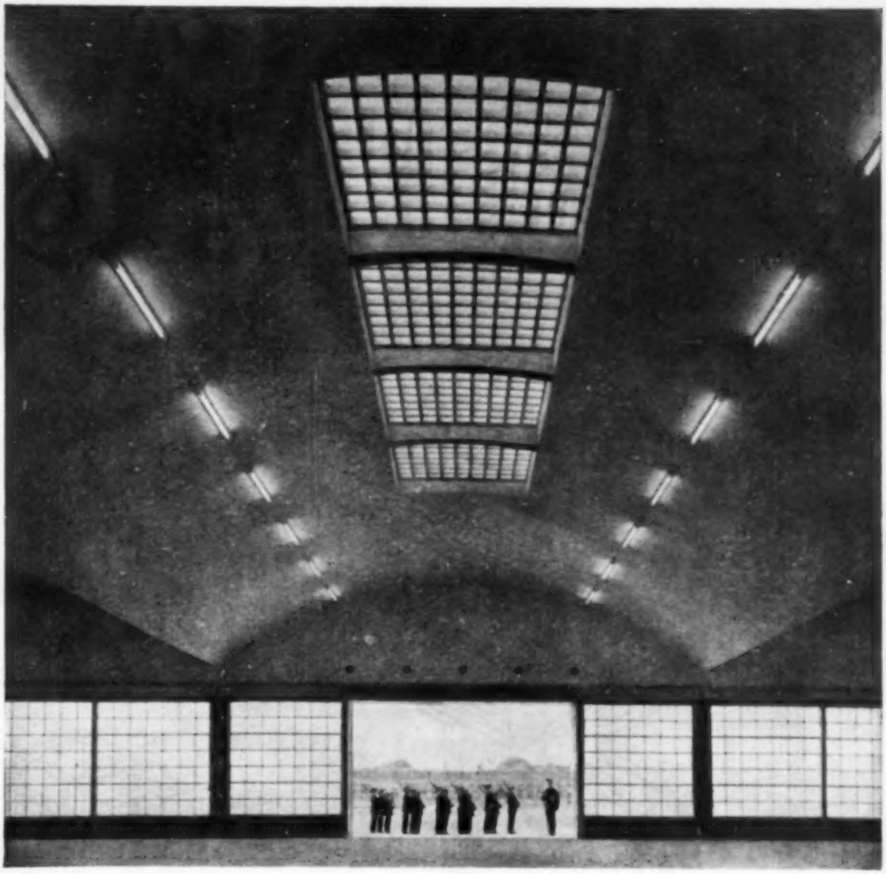
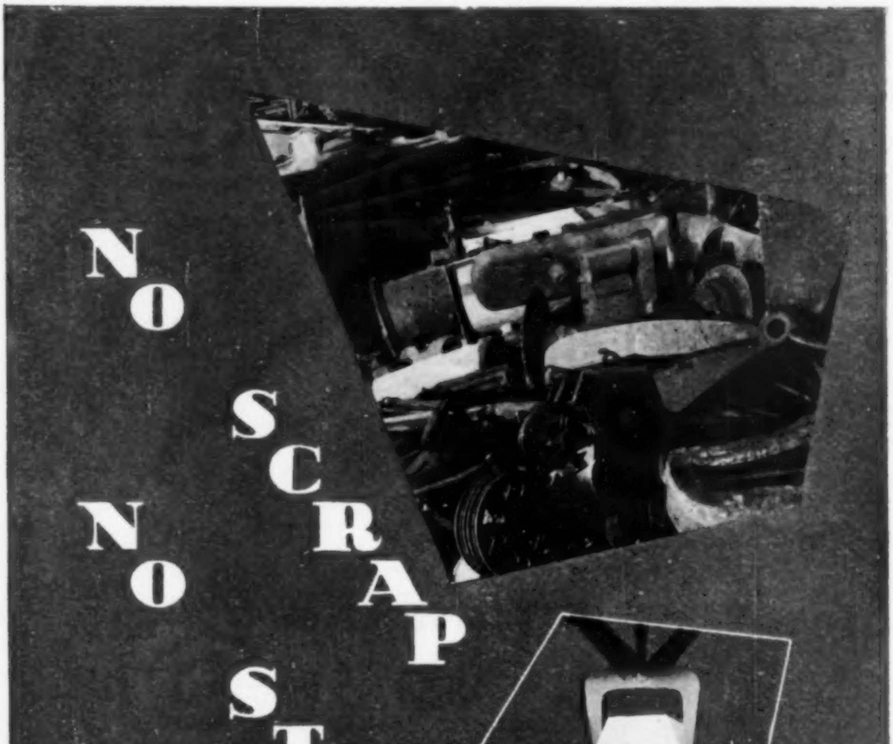


Reinforcement in position for a bow girder for a building in Uxbridge Road, Ealing. *Architect :* Mr. H. E. Davie, A.R.I.B.A. *Consulting Engineer :* Mr. F. J. Samuely, B.Sc. (Eng.) Lond., A.M.I.C.E., M.I.Struct.E., F.I.A.S., M.I.W. *Contractors,* Tersons, Ltd.



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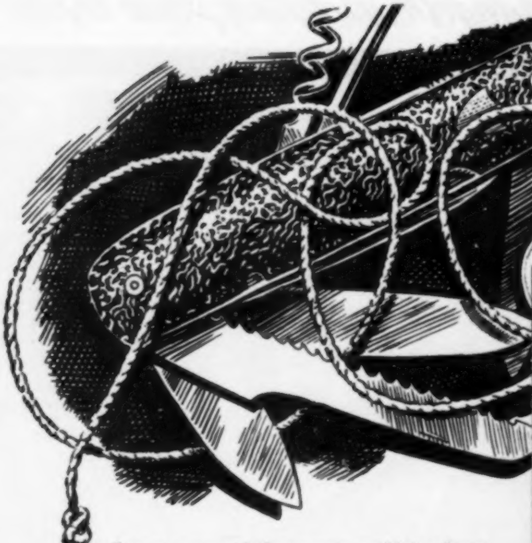

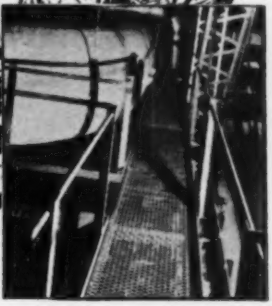

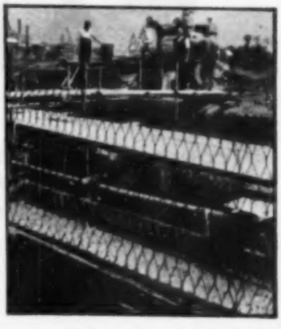

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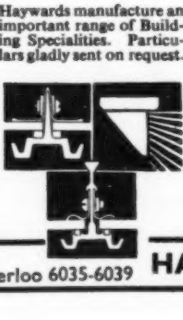

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Volume XLVII. No. 2. LONDON, FEBRUARY, 1952.

EDITORIAL NOTES

The Supply of Reinforcement Bars.

WHEN a raw material is scarce it is perhaps natural that it should be used by the manufacturers, or by a Board which controls a nationalised industry and may be anxious to set up record production figures, in the easiest or the most profitable manner. This seems to be the case with steel for use in buildings, for it is absurd that more priority is not given to the shapes which will give the greatest volume of construction for the least amount of such a scarce material. The case for a larger allocation of steel for concrete reinforcement has been stated in a forceful manner by the secretary of the Reinforced Concrete Association in a letter to "The Times," in which he points out that a reinforced concrete structure requires only one-third to one-half of the steel required in a comparable steel structure, and instances a recent case of a multi-story block of flats requiring 880 tons of structural steel which had been built with only 330 tons of reinforcement bars. He also points out that one ton of high-carbon steel in prestressed concrete may save ten or twelve tons of structural sections. The argument that, because the shortage affects all the steel-consuming industries, an increased allocation for one usage can be made only at the expense of another is fallacious. The diversion of billets from structural sections to reinforcement bars would not, the letter continues, mean that any structure could not be built, because two or three structures could, in fact, be built with the same amount of steel. The reinforced concrete industry could well use additional steel at the rate of 10 per cent. a year, and the rolling capacity of the British mills is more than ample for the purpose. An increase of 10 per cent. in the output of bars is equivalent to about 25,000 tons a year. A reduction of 25,000 tons a year in the output of structural sections of the shape used in building and civil engineering work would represent little more than 1 per cent., a change-over that could scarcely disturb the balance of industry to the maintenance of which so much importance appears to be ascribed. Such a change must eventually follow the normal progress of structural design and might well be accelerated to meet the present emergency. Nevertheless, the weekly average output of bars during 1951 has been nearly 10 per cent. less than in 1950 and a considerable quantity is being imported to enable essential works to be carried on. The letter concludes by pointing out that wire for prestressed concrete is also scarce.

It has been calculated that the amount of steel used in the form of girders

in the foundation of a large building now under construction for one of the nationalised industries would be sufficient for the whole of the superstructure as well as the foundation if both were built in reinforced concrete, and the consumption of steel in unnecessarily large quantities where reinforced concrete would be equally suitable is common. This unnecessary use of steel is sometimes due to the fact that it is easier to get steel sections than reinforcement bars in the required sizes, which, together with the high prices that have been paid for bent bars, are clear proofs that the allocation of steel for reinforcement bars is too small.

This claim that more steel should be available in the form in which it can be used most economically goes much farther and is likely to have more influence than the suggestion of the Council of the Association (reported in our last number) that the working stress of 18,000 lb. per square inch normally permitted in mild steel reinforcement might safely be increased and the amount used in reinforced concrete structures thus reduced. The Council of the Association expresses the view that a working stress of 20,000 lb. per square inch might safely be adopted in bars of ⅝-in. diameter and less, except in the design of liquid-retaining and other special structures. It is the opinion of the Council that standard specifications and codes of practice, which are not mandatory, ought not to be varied to meet temporary emergencies, but that in the case of mandatory regulations, such as those of the London County Council, applications for waivers to permit the suggested higher stress be sympathetically considered. An increase in the permissible stress in bars of ⅝-in. diameter and less would not effect much saving of steel in reinforced concrete, because such bars are mainly used in slabs and walls and generally form only a small part of the total weight of reinforcement in a structure. Equal or greater saving can be effected by adopting a lower maximum compressive stress in the concrete, thereby decreasing the amount of reinforcement required at the expense of a little more concrete and without decreasing the factor of safety associated with the tensile stress of 18,000 lb. per square inch. The statement of the Association mentions that the question of whether the tensile stress permitted in bars larger than ⅝ in. can be raised is being considered, but here again it is often possible by slightly increasing the depth of a beam or the cement content of the concrete in a column to effect a worthwhile saving of reinforcement without decreasing the factor of safety. There is, of course, a great demand for cement as well as for steel, but this is already being largely supplied by new cement works coming into operation. Also the fact that the principal raw materials for cement are produced in this country, and are not largely imported as is the case with steel, is an important consideration.

It may well be that some difficulty will be experienced in reconciling two points in the suggestion of the Association, namely (i) that the recommendations of the present codes and regulations should not be varied and that the suggested higher tensile stress should be permitted in emergencies only, and (ii) that the Association considers that the higher stress can be permitted with safety. If a structure is designed for a higher stress it follows that it has a smaller factor of safety than one designed for a lower stress. If the smaller factor of safety is sufficiently large, then the codes and regulations should be altered and the higher stresses permitted generally. If the higher stress does not result in a sufficiently large factor of safety, then the existence of a temporary emergency is not a good reason for building permanent structures that are not sufficiently safe.

The Design of An Unusual Bow Girder.

By V. A. MORGAN, M.Eng., A.M.Inst.C.E.

A REINFORCED concrete bow girder (*Figs.* 1 and 2) 30 ft. long on its centre line and carrying columns supporting upper floors has been built at the level of the first floor of a building in Uxbridge Road, Ealing. The building is a reinforced concrete frame structure, designed in accordance with British Standard Code of Practice No. 114, with concrete floors and brick walls. The mixture of the concrete in the girder is 1 : 1 : 2, and as consolidation was by mechanical vibration the maximum permissible compressive stress in bending is 1650 lb. per square inch.

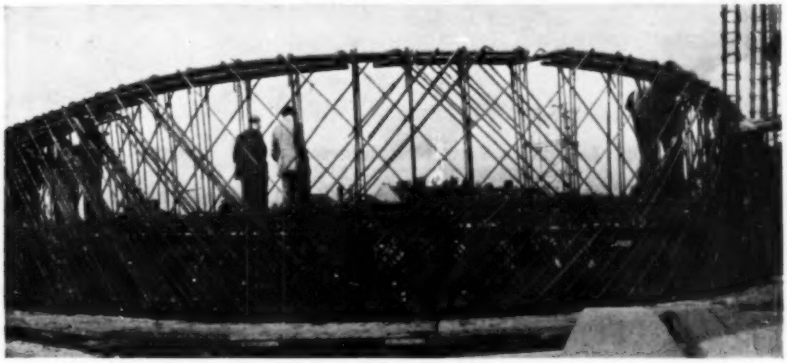

Fig. 1.—Reinforcement of Girder.

The crushing strength of 6-in. cubes at seven days was about 7000 lb. per square inch for rapid-hardening Portland cement concrete having a slump of 1 in. to 2 in.

Calculation of Twisting Moments.

The girder comprises a central quadrant P1—P2 (*Fig.* 3) of 9 ft. 3 in. radius and straight lengths B—P1 and C—P2 of 7 ft. 8 in. tangential to the curve at each end. The design is therefore more complex than for a girder that is curved throughout its length. Two columns F and G on the quadrant cause large twisting moments on the girder. Two other columns E and H are carried on the straight parts. The ordinary bending moments and shearing forces are resisted by longitudinal and inclined bars and vertical binders. The twisting moments are resisted by helical binding. Since the theory of twisting of rectangular reinforced concrete beams is incomplete, the design is based on the published results of experiments. The loads on the girder are considered to be concentrated at E, F, G, and H, although they are partly distributed. Three limiting cases are considered in the following for the calculation of the moments.

CASE I.—GIRDER RIGIDLY FIXED AT SUPPORTS.—The girder is assumed to be rigidly fixed at the supporting columns B and C (*Fig.* 3). Consider half the girder BK having a bending moment M_K applied at K equal to the bending moment at the mid-point of the girder. Because of symmetry there is no twisting

Fig. 2.—Inside View of Reinforcement of Girder.

moment or shearing force at K. At any point Q on the curve, the bending moment M_Q and twisting moment T_Q are given by

$$M_Q = M_K \cos\theta - W_F R \sin(\theta - \phi) \quad . \quad . \quad . \quad . \quad (1)$$

$$T_Q = M_K \sin\theta - W_F R[1 - \cos(\theta - \phi)] \quad . \quad . \quad . \quad (2)$$

The strain-energy equation for the semi-quadrant PK is

$$\frac{\partial U}{\partial M_K} = \frac{1}{EI}\left[\int_0^{\frac{\pi}{4}} M_K R \cos^2\theta . d\theta - \int_\phi^{\frac{\pi}{4}} W_F R^2 \sin(\theta - \phi)\cos\theta . d\theta\right]$$
$$+ \frac{1}{CJ}\left\{\int_0^{\frac{\pi}{4}} M_K R \sin^2\theta . d\theta - \int_\phi^{\frac{\pi}{4}} W_F R^2 \sin\theta[1 - \cos(\theta - \phi) \, d\theta\right\},$$

where C is the torsional rigidity and J is the equivalent polar moment of inertia. Integrating and substituting $W_F = 48{,}000$ lb., $\phi = 17$ deg., and $R = 9{\cdot}25$ ft.,

$$\frac{\partial U}{\partial M_K} = \frac{1}{EI}(5{\cdot}95 M_K - 380{,}000) + \frac{1}{CJ}(1{\cdot}325 M_K - 45{,}000) \quad . \quad (3)$$

In the straight part BP, for point Q at distance x from P

$$M_Q = \frac{M_K}{\sqrt{2}} - W_F(x + l_F) - W_B(x - l_B) \quad . \quad . \quad . \quad (1a)$$

$$T_Q = \frac{M_K}{\sqrt{2}} - W_F h_F \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2a)$$

The strain-energy equation for BP, with numerical values from *Fig.* 3 substituted, is

$$\frac{\partial U}{\partial M_K} = \frac{1}{EI}\left[\int_0^{7{\cdot}67} \frac{M_K}{2} dx - \int_0^{7{\cdot}67} \frac{48{,}000}{\sqrt{2}}(x + 4{\cdot}35)\, dx - \int_1^{7{\cdot}67} \frac{62{,}000}{\sqrt{2}}(x - 1)\, dx\right]$$
$$+ \frac{1}{CJ}\left[\int_0^{7{\cdot}67} \frac{M_K}{2} dx - \int_0^{7{\cdot}67} \frac{51{,}800}{\sqrt{2}} dx\right]$$
$$= \frac{1}{EI}(3{\cdot}82 M_K - 3{,}110{,}000) + \frac{1}{CJ}(3{\cdot}82 M_K - 280{,}000) \quad . \quad . \quad (3a)$$

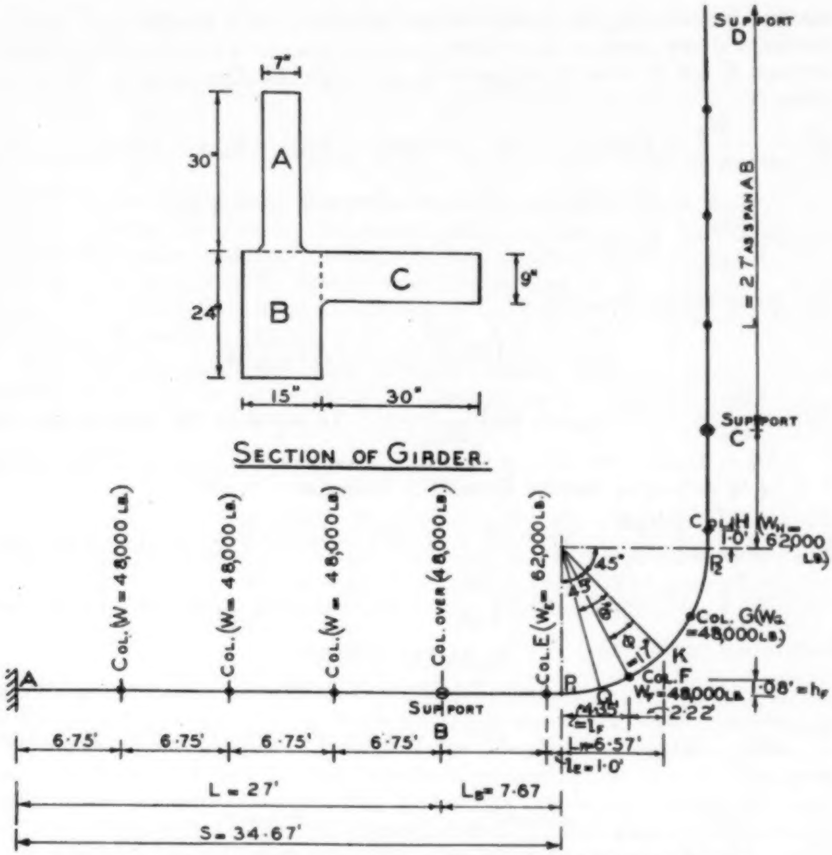

Fig. 3.—Plan and Cross Section of Girder and Adjoining Beams.

Adding (3) and (3*a*), for the girder BK,

$$\frac{\partial U}{\partial M_K} = \frac{1}{EI}(9{\cdot}77M_K - 3{,}490{,}000) + \frac{1}{CJ}(5{\cdot}145M_K - 325{,}000) = 0 \quad . \quad (3b)$$

Assuming $\dfrac{EI}{CJ} = 10$ (as discussed later), $M_K = 110{,}000$ ft.-lb. From (2*a*), the twisting moment on BP is 26,000 ft.-lb. Differentiating (2) and equating to zero defines the position of the point of maximum twisting moment as $\theta = 30$ deg. approximately. Substituting in (2), the greatest twisting moment is 43,500 ft.-lb. From (1*a*), the bending moment M_B at B is $-$ 912,200 ft.-lb.

Case II.—Girder Continuous over Supports.—The girder is considered to be the middle span of a symmetrical three-span beam the ends of which are assumed to be fixed at the columns at A and D. Only half the beam need be analysed, the origin being at K. The indeterminate reaction R_B at B and the

bending moment M_K at K are to be calculated. The bending and twisting moments at any point on the straight part at a distance x from the perpendicular through K are M_x and T_x respectively and, with the dimensions in *Fig.* 3, are given by

$$M_x = -\frac{M_K}{\sqrt{2}} + 48{,}000(x - 2{\cdot}22) + 62{,}000(x - 7{\cdot}57) - R_B(x - 14{\cdot}24) \quad . \quad (1b)$$
$$+ 48{,}000[(x - 20{\cdot}99) + (x - 27{\cdot}74) + (x - 34{\cdot}49)]$$
$$T_x = -\frac{M_K}{\sqrt{2}} + 51{,}800.$$

The strain-energy equation is

$$\frac{\partial U}{dR_B} = \frac{1}{EI}\int M_x \frac{\partial M_x}{\partial R_B} dx + \frac{1}{CJ}\int T_x \frac{\partial T_x}{\partial R_B} dx.$$

Now, $\frac{\partial M_x}{\partial R_B} = -(x - 14{\cdot}24)$, and $\frac{\partial T_x}{\partial R_B} = O$. To simplify the calculations, let $L = AB = 27$ ft., and $X = x - 14{\cdot}24$. Therefore $dX = dx$, and the limits $L + 14{\cdot}24$ and $14{\cdot}24$ become L and O. Therefore

$$\frac{\partial U}{\partial R_B} = \frac{1}{EI}\left\{\int_0^L \left[\frac{M_K X}{\sqrt{2}} - 48{,}000(X + 12{\cdot}02)X - 62{,}000(X + 6{\cdot}67)X + R_B X^2\right] dX\right.$$
$$\left. -48{,}000\left[\int_{6\cdot75}^L (X - 6{\cdot}75)X\,.\,dX + \int_{13\cdot5}^L (X - 13{\cdot}5)X\,.\,dX + \int_{20\cdot25}^L (X - 20{\cdot}5)X\,.\,dX\right]\right\} = 0$$

Integrating, substituting $L = 27$ ft., and transposing,

$$R_B = 214{,}330 - 0{\cdot}0393 M_K \quad . \quad . \quad . \quad . \quad (4)$$

Also $\frac{\partial M_x}{\partial M_K} = \frac{\partial T_x}{\partial M_K} = -\frac{1}{\sqrt{2}}$. Let $S = 34{\cdot}67$ ft. and $Y = x + 6{\cdot}57$; then $dY = dx$.

$$\frac{\partial U}{\partial M_K} = \frac{1}{EI}\left\{\int_0^s \frac{1}{2} M_K dY - \frac{48{,}000}{\sqrt{2}}\int_0^s (Y + 4{\cdot}35) dY - \frac{62{,}000}{\sqrt{2}}\int_1^s (Y - 1) dY\right.$$
$$+ \frac{R_B}{\sqrt{2}}\int_0^L X\,.\,dX - 48{,}000\left[\int_{6\cdot75}^L (X - 6{\cdot}75) dX + \int_{13\cdot5}^L (X - 13{\cdot}5) dX\right.$$
$$\left.\left. + \int_{20\cdot25}^L (X - 20{\cdot}25)\, dX\right]\right\} = \frac{24{\cdot}5}{EI}\left(\frac{M_K}{\sqrt{2}} - 2{,}508{,}500 + 10{\cdot}51 R_B\right)$$

Substituting for R_B from (4), $EI\frac{\partial U}{\partial M_K} = 7{\cdot}2 M_K - 6{,}240{,}000 \quad . \quad . \quad . \quad (3c)$

The corresponding equation for the c urved part of the girder is given by the first term in (3) in Case I, and by (3*c*).

$$\frac{\partial U}{\partial M_K} = \frac{1}{EI}(13{\cdot}15 M_K - 6{,}620{,}000). \quad . \quad . \quad . \quad (3d)$$

Also,

$$\frac{\partial U}{\partial M_K} = \frac{1}{CJ}\int_{6\cdot57}^{S+6\cdot57} \frac{1}{2} dx - 51{,}800\int_{6\cdot57}^{S+6\cdot57} \frac{1}{\sqrt{2}} dx = \frac{1}{CJ}(17{\cdot}35 M_K - 1{,}270{,}000) \; . \quad (3e)$$

Adding the second term in (3) to (3*e*) and combining with (3*d*), the total strain energy is given by

$$\frac{\partial U}{\partial M_K} = \frac{1}{EI}(13 \cdot 15 M_K - 6{,}620{,}000) + \frac{1}{CJ}(18 \cdot 675 M_K - 1{,}315{,}000) = 0. \quad (3f)$$

Substituting $\frac{EI}{CJ} = 10$, $M_K = 99{,}000$ ft.-lb. The twisting moment on the straight part BP is determined from (2*a*) in Case I and is equal to 18,200 ft.-lb. The greatest twisting moment on the curved part is 38,230 ft.-lb. and occurs at $\theta = 28\frac{1}{2}$ deg. approximately. Substitution in (4) gives $R_B = 210{,}450$ lb., or, with the load of 48,000 lb. directly over B, the load on the column at B is 258,450 lb. From (1*b*), $M_B = -920{,}000$ ft.-lb. and $M_A = -140{,}000$ ft.-lb. Also $R_A = 91{,}200$ lb.

CASE III.—STRAIGHT PARTS OF GIRDER FREE TO ROTATE.—The girder is assumed to be free to rotate along the straight parts; therefore there is no twisting moment on these parts. It is, however, restrained in bending. The basic formulæ are

$$M_Q = -M_K \cos\theta + W_P R \sin(\theta - \phi)$$

$$T_Q \text{ (on curve only)} = W_P R[1 - \cos(\theta - \phi)] - M_K \sin\theta.$$

Substituting $\theta = 45$ deg., $\phi = 17$ deg., $R = 9 \cdot 25$ ft., and $W_P = 48{,}000$ lb., and equating T_Q to zero (for $\theta = 45$ deg.), M_K is 73,600 ft.-lb. The greatest twisting moment is 27,600 ft.-lb. and occurs at $\theta = 43$ deg. approximately.

Helical Binding.

The girder is designed to resist a maximum twisting moment of 480,000 in.-lb. on the curved part and 312,000 in.-lb. on the straight part, these values being intermediate between those calculated in the foregoing and less than those in Case I because conditions are less rigorous than assumed in Case I. The floor slab is monolithic with the girder (*Fig.* 3) and this reduces the twisting moment; but this effect is neglected, although a small amount of twisting moment is assumed to be resisted by a slab of limited width. Most of the twisting moment is, however, resisted by the lower part of the girder, but some is also resisted by the upper part. Continuous helical binding inclined at 45 deg. in the lower part, and similar binding extending from the upper part to the bottom of the lower part, is provided to resist the twisting.

The helical binding and longitudinal reinforcement are designed to resist torsion in accordance with the data given by Professor W. T. Marshall and Mr. N. R. Tembe,[1] the modulus of rigidity being $1 \cdot 2 \times 10^6$ lb. per square inch. The formula used for the torsional resistance T' of a section composed of several rectangles is that given by Mr. Leslie Turner and Mr. V. C. Davies,[2] that is

$$T' = \frac{4A}{PD}(R_1 + R_2 + R_3 + \ldots) \quad . \quad . \quad . \quad (5)$$

where A and P are the area and perimeter of the section, R_1, R_2, etc., are the torsional resistances of the separate rectangles, and D is the diameter of the largest inscribed circle.

The torsional rigidity JC of, and the maximum shearing stress s' in,

rectangular sections are calculated from the formulæ given by Timoshenko and Lessells,[3] namely,

$$J = K_2 b^3 d \text{ and } s' = \frac{T}{K_1 b^2 d}.$$

For rectangles of the sizes shown in *Fig.* 3, the values of K_1, K_2, J, and s' are given in the following :

Rectangle	d in.	b in.	$\frac{d}{b}$	K_1	K_2	J in.^{4}	T in.-lb.	s' lb. per sq. in.
A	30	7	4·286	0·286	0·285	2930	55,800	133
B	24	15	1·6	0·235	0·203	16,450	313,000	237
C	30	9	3·3	0·272	0·268	5850	111,200	168
			Total rigidity = 25,230 in.^{4}			Total =	480,000	

The twisting moments T are calculated by proportion from the values of J. The factor $\frac{4A}{PD}$ in (5) is equal to 1·13 in the present case, showing that the torsional resistance of the combined section is 13 per cent. greater than the sum of the separate resistances, and to this extent at least the calculations are conservative.

At the point of maximum twisting moment on the curved part, the vertical shearing stress is about $\frac{48{,}000}{0{\cdot}83 \times 46{\cdot}5 \times 7} = 178$ lb. per square inch, which when added to s' for section A and B results in shearing stresses of 311 lb. and 415 lb. per square inch respectively. Because this stress is so high, helical reinforcement is provided throughout the curved and straight parts of the girder. Transverse beams at the columns B and C convert the torsion at these points into ordinary bending moments which are resisted by the beams.

Torsional stresses are greatest at the mid-point of the sides of a rectangular section, and cracks due to excessive diagonal tensile stress commence at these points and proceed diagonally. Mr. H. J. Cowan[4] has shown that torsion is initially resisted by the concrete alone, but, as the torsion increases, the concrete cracks and the helical binding comes into play, the stress in the binding being proportional to the twisting moment. Rausch's formula,

$$A_b = \frac{pT}{2\sqrt{2}.tF} \quad . \quad . \quad . \quad . \quad . \quad (7)$$

is used to design the helical binding, A_b being the cross-sectional area of the bar of which the binding is formed, p the pitch of binding, t the tensile stress in the binding, and F the area enclosed by the binding. Thus in section B,

$$\frac{A_b}{p} = \frac{313{,}000}{2\sqrt{2} \times 18{,}000 \times 13 \times 21} = 0{\cdot}0226,$$

which is given by $\frac{3}{8}$-in. binding at 5-in. pitch. Actually, in a length of 68 in., eighteen turns of the helical binding are provided. The binding starts at the

centre of the curved part of the girder and is wound in a clockwise direction at 45 deg. towards column B and in an anticlockwise direction towards column C. Thus as the girder tends to twist in the opposite direction, the coils of the binding are tightened and resist the torsional forces and compress the concrete core. At joints in the binding the bars overlap and are welded. A similar calculation shows that $\frac{3}{8}$-in. helical binding at 12-in. pitch as provided in section A is sufficient. The binding required in section C is negligible; therefore $\frac{3}{8}$-in. bars at 6-in. centres are provided in both directions in the top of the slab in addition to the ordinary reinforcement of $\frac{1}{2}$-in. bars at 6-in. centres in the bottom. The vertical shearing forces on the girder are resisted by vertical binding and inclined bars in the ordinary way.

The rigidity of the entire section of the girder, with $J = 1{\cdot}13 \times 25{,}230 = 28{,}500$ in.^{4}, $C = 1{\cdot}2 \times 10^6$ lb. per square inch, $E = 3 \times 10^6$ lb. per square inch, and $I = 130{,}000$ in.^{4} for A and B plus 1820 in.^{4} for C, is given by

$$\frac{EI}{CJ} = \frac{3 \times 10^6 \times 131{,}820}{1{\cdot}2 \times 10^6 \times 28{,}500} = 11{\cdot}5.$$

Because of continuity with the floor slab, a value of 10, as assumed in the calculations, is reasonable.

Notes on the Stresses and Action of the Binding.

Tensile Stress in the Binding.—If b' and d' are the dimensions of the core enclosed by the helical binding (*Fig.* 4), consider a part of the girder of length d'. The effect of the twisting moment is equivalent to that of two forces P' acting at the ends of the diagonal AB and two equal and opposite forces at the ends of diagonal CD. These forces cause, across diagonal CD, tensile forces which are resisted by the concrete or, if the concrete cracks, by the helical binding which crosses the diagonal at right-angles. The twisting moment T is $P'd'$, and the

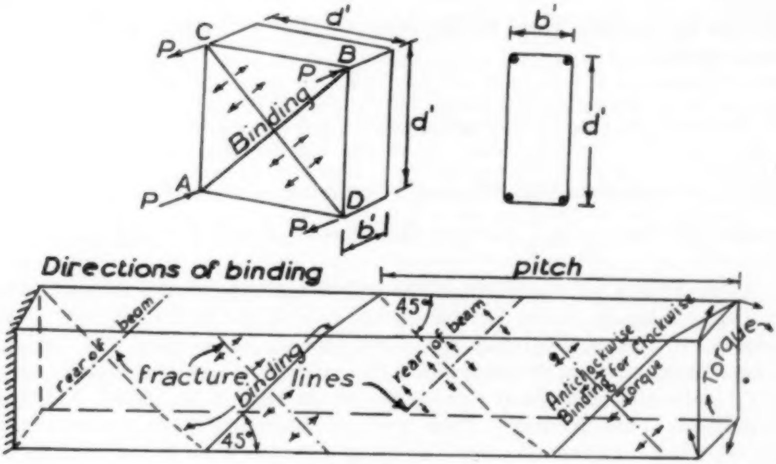

Fig. 4.

bending moment at the diagonal is $\frac{2P'.d'\sqrt{2}}{4}$, that is $\frac{T}{\sqrt{2}}$. The average tensile force in the binding is $\frac{T}{\sqrt{2}}.\frac{1}{b'}$ for the longer side and similarly $\frac{T}{d'\sqrt{2}}$ for the shorter side. Hence the average tensile stress is $\frac{1}{4}.\frac{2T}{A_b\sqrt{2}}\left(\frac{1}{b'}+\frac{1}{d'}\right)$, and therefore for one system only of helical binding the maximum stress t is $\sqrt{2}$ times the average stress, that is, $t = \frac{T\sqrt{2}}{4\sqrt{2}b'd'A_b}[2(d' + b')]$.

If there are n systems interwoven, $t = \frac{T\sqrt{2}}{4\sqrt{2}b'd'A_b}\left[\frac{2(d' + b')}{n}\right]$; but $\frac{2(d' + b')}{n}$ is the horizontal spacing of the binding, or $\sqrt{2}$ times the pitch p. Therefore $t = \frac{Tp}{2\sqrt{2}b'd'A_b}$, which is Rausch's formula.

SHEARING STRESS IN THE CONCRETE.—If the resistance of the concrete, including that outside the binding, is considered, the section is d by b, and the forces P (corresponding to P') act at distances $\frac{d}{6}$ from the top and bottom edges of the girder. The twisting moment T is $\frac{2}{3}dP$. The average stress s_a is the bending moment caused by P on the diagonal $\left(\frac{2P \times \frac{2}{3}d\sqrt{2}}{4}\right)$ divided by the modulus of the diagonal plane $\left(\frac{b^2d\sqrt{2}}{6}\right)$, that is $\frac{3T}{b^2d}$.

The maximum stress s_1 (diagonal shear) is $1\frac{1}{2}s_a$, that is $s_1 = \frac{9}{2}.\frac{T}{b^2d}$, which is the formula given by Mr. Seely.[5] The writer has developed the empirical formula

$$s_1 = \frac{T}{\left(0{\cdot}188 + 0{\cdot}01\sqrt{29\frac{d}{b} - 25}\right)b^2d},$$

which is more rigorous than Mr. Seely's and agrees very closely with St. Venant's classical solution[6] for $\frac{d}{b}$ between the common limits of 1 and 4.

COMPRESSION OF THE CORE.—The helical binding, besides directly resisting the diagonal tensile forces, compresses the core of the girder, and the resulting compressive stresses counteract the tensile stresses in the core. As the binding changes direction at each longitudinal bar, a total pressure of $2T_b \sin 120$ deg. ($= T_b$) is directed inwards at right-angles to the edge of the girder, T_b being the tensile force in the binding. There are also shearing stresses which counteract each other but compress the core in so doing and contribute to the ultimate resistance of the girder.

Professor Marshall and Mr. Tembe [1] observed that failure in torsion seems to be due to destruction of bond and disintegration of the concrete. These effects may be caused partly by the large bearing pressure exerted at changes of direction of the binding. Because of the possibility of failure of bond accentuated by crushing, helical binding should be continuous ; where joins are unavoidable the bars forming the binding should be welded, as was done in the girder described in this article.

The architect for the building is Mr. E. H. Davie, A.R.I.B.A., and the consulting engineer Mr. F. J. Samuely. The contractors are Messrs. Tersons, Ltd. The reinforcement was supplied and fixed by the Rom River Co., Ltd.

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(2) L. Turner and V. C. Davies. Plain and Reinforced Concrete in Torsion. Institution of Civil Engineers Selected Paper No. 165 (1934).
(3) S. Timoshenko and J. M. Lessells. "Applied Elasticity" (1925).
(4) H. J. Cowan. Tests of the Torsional Strength and Deformation of Rectangular Reinforced Concrete Beams. "Concrete and Constructional Engineering," February, 1951.
(5) F. B. Seely. "Advanced Strength of Materials."
(6) A. E. H. Love. "Mathematical Theory of Elasticity." Vol. 1.
(7) E. Rausch. "Bewehrung des Eisenbetons gegan Verdrehung und Abscheren."

British Standard for Concrete Poles.

The revised British Standard for reinforced concrete poles for electrical transmission lines (No. 607, 1951. Price 2s. from the British Standards Institution) deals with design and testing. It is left to the manufacturers to determine the cross-sectional dimensions. Six classes of pole capable of resisting ultimate transverse loads of 625 lb. to 3500 lb. are considered. The poles must be designed so that failure is due to permanent stretching of the reinforcement and to failure of the concrete in compression. The concrete must be composed of 1 part of Portland cement to not more than 4½ parts of aggregate. The water-cement ratio must not exceed 0·55. The thickness of concrete over the reinforcement must be not less than the greatest size of the aggregate plus 1/16 in. and not less than ¾ in., except that for poles made by a centrifugal process a minimum of ½ in. is allowed. Where bars are lapped, the lap must not be less than 39 times the diameter of the smaller bar.

Poles may be tested in a horizontal or a vertical position, and must be rigidly supported at the butt for a distance equal to the depth of planting. The load is applied 2 ft. from the top of the pole and the deflection is measured at loads increasing by increments of 10 per cent. up to 50 per cent. of the minimum ultimate load specified. The load is removed and the permanent set measured. The load is then again applied gradually up to 60 per cent. of the ultimate load, when it is again removed and the permanent set measured. This operation is repeated, the load being increased successively by increments of 10 per cent. of the specified ultimate load up to 80 per cent. and then by increments of 5 per cent. until failure occurs.

Anchoring Wires in Prestressed Concrete.

THE Franki-Smet method of anchoring the wires in prestressed concrete comprises conical keys (*Figs.* 1 and 2) which allow each wire to be anchored separately, although any number of wires can be tensioned at one time by an hydraulic jack. Generally, however, the jack is capable of stretching one, two, or three wires at a time so that the tensile force in each wire in a cable is as far as possible equal. The anchoring of a cable comprising ten 5-mm. wires is shown in *Fig.* 1 and twelve 7-mm. wires in *Fig.* 2. In passing through perforated cups (A1) the wires are spaced uniformly in the cable (as at A2), and are splayed out at a small angle to pass through a thick steel anchor-plate (A3) which bears against a steel annular ring at the end of the beam. The anchor-plate is perforated with one hole for each wire. The conical keys which secure the wires in the plate are segmental in cross section. A central hole in the plate allows the jack to be centred while the wires are being stretched, and permits grout to be injected into the cable.

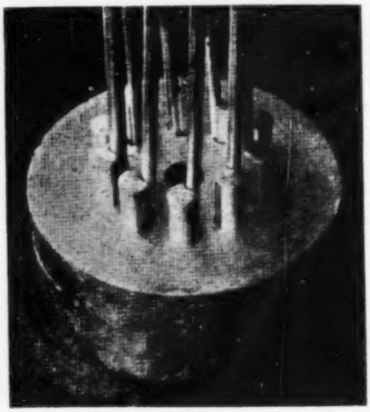


Fig. 1.

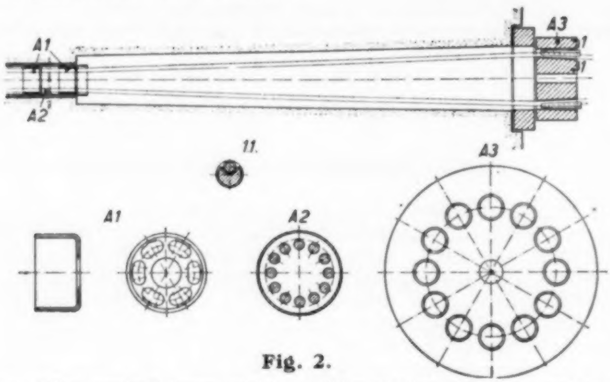


Fig. 2.

Successful tests have been made at Liége University on anchorages of this type and on beams incorporating these anchorages, which are used in a bridge of 113-ft. span near Ghent. The foregoing notes and accompanying illustrations are from "La Technique des Travaux".

Precast Concrete in Buildings.

PRECAST reinforced concrete slabs, beams, and frames are used in the buildings for the new technical college at Hatfield. The structural members in the building for the engineering department (*Fig.* 1) and gymnasium are precast three-hinge frames of 33 ft. span, spaced at 11 ft. and 10 ft. centres respectively, and precast purlins. The roof covering of the engineering department is asbestos-cement sheeting, but the roof of the gymnasium is of unusual design incorporating precast slabs as described later. Precast frames of 44 ft. span and 48 ft. high are provided in the assembly hall (*Fig.* 2), the roof of which is of precast slabs. Suspended floors and flat roofs are constructed of trussed beams (*Fig.* 3) and precast slabs.

The proportions of the concrete in the precast members are 1 : 1½ : 3 by volume, the greatest size of the aggregate being ¾ in. The concrete was consolidated by vibration and the maximum compressive stress is 1375 lb. per square inch. The reinforcement is plain round mild steel bars in which the maximum tensile stress is 18,000 lb. per square inch. The large frames were cast on the site, but the slabs and smaller members were cast in a factory.

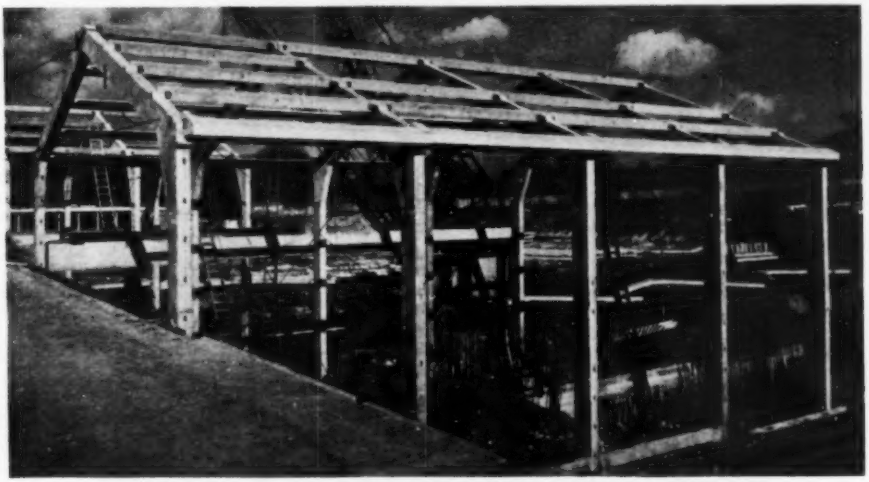


Fig. 1.—Construction of Engineering Building.

Trussed Beams and Precast Slabs.

The beams for suspended floors and flat roofs each comprises a precast reinforced concrete compression boom and a mild steel tie the shape of which is maintained by two concrete struts as shown in *Fig.* 3. The beams are at 5 ft. 6 in. centres and support precast slabs which act as permanent shuttering for a concrete slab cast in situ. Reinforcement projects above the top boom of the beams to bond with the cast-in-situ concrete. For beams spanning 27 ft. 6 in., the top boom is 7 in. wide and 3½ in. deep and is designed primarily to resist compressive stresses due to the weights of the beam, the precast slabs, and wet cast-in-situ concrete. When the cast-in-situ concrete has hardened, the composite beam resists the dead load and the superimposed loads. For floor beams, the tie is 3 in. by 1¼ in., and for roof beams 3 in. by ⅝ in. The maximum tensile stress in the tie occurs when the dead and superimposed loads are acting, and does not exceed 18,000 lb. per square inch. The struts are reinforced, the bars projecting above the top of the boom forming eyes to which two-

point suspension tackle was attached for lifting the beam into position.

The beams were cast on the steel frames shown in *Fig.* 4, which comprised an inverted steel channel (which formed the bottom of the mould) supported on two strutted end-posts also of steel channels. The steel tie and concrete struts were placed in position below the channel, and steel-angles to form the sides of the mould were attached to cleats projecting from the channel. When the concrete had set the angles were removed and the beam left on the channel until the concrete had hardened sufficiently to enable it to be removed without damage due to excessive compression in the concrete or to lack of bond at the anchorage of the tie. At each end of the beam (*Fig.* 5) a steel plate engages a perforated lug projecting from a steel bracket on the head of the column supporting the beam which is then secured by wedges tapered in two planes.

The precast slabs are inverted troughs 3 in. deep overall, 18 in. wide, and 5 ft. 1 in. long, and the concrete is 1 in. thick. Each slab weighs about 100 lb. Two or more holes are formed in the slab for lighting fittings.

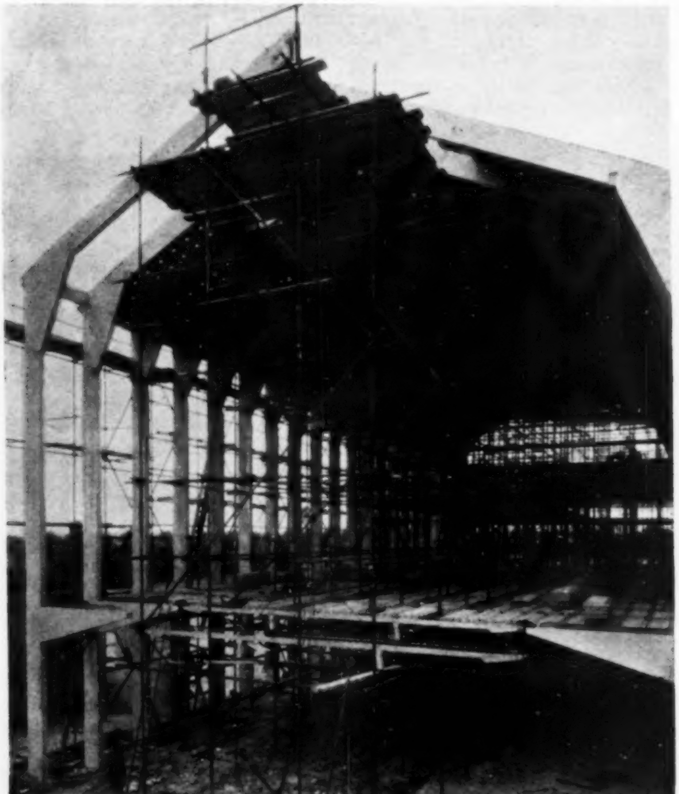


Fig. 2.—Frames and Beams of Assembly Hall.

Gymnasium Roof.

The posts of the three-hinge frames supporting the roof of the gymnasium are 18 ft. 3 in. high to the eaves and are 15 in. by 7 in. in cross section. As it was

not permitted for architectural reasons to provide a haunch at the junction of the post and rafter, it is necessary for the horizontal reaction at the bottom of the post to be as small as possible so that the bending moment at the head of the post is also small. This condition is obtained by designing the sloping roof as a thin-web girder resisting the outward forces in its own plane. The span of the girder is 70 ft. and the reaction at each end is provided by a horizontal tie-beam 11⅝ in. deep and 5½ in. wide extending at eaves level across the end frames. The roof covering (*Fig.* 6) comprises precast slabs, similar to those in the flat roofs, supported on precast purlins at about 6 ft. 1 in. centres and screeded with 1 in. cast-in-situ 1 : 1½ : 3 concrete which is covered with cedar shingles. The underside of the precast slabs is lined with glass-wool and fibre-board. The weight of the roof is about 40 lb. per square foot. The concrete screed, the

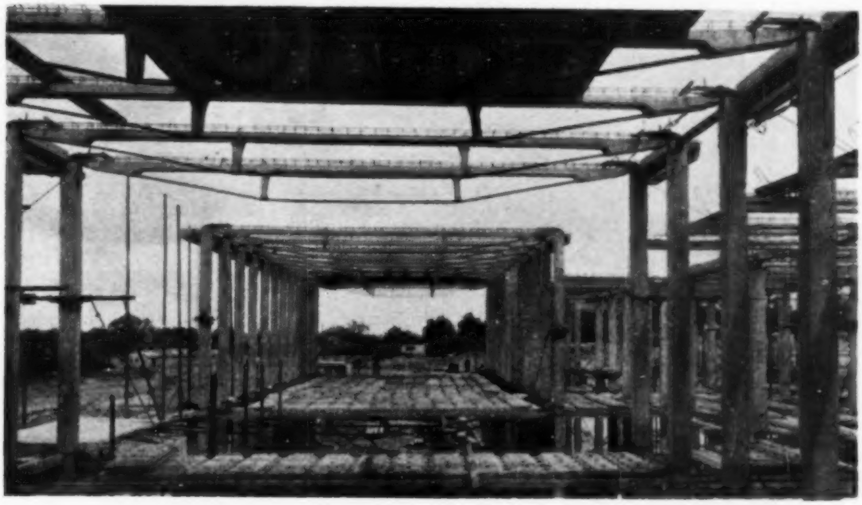


Fig. 3.—Beams and Floor Slabs.

Fig. 4.—Casting Trussed Beams.

maximum size of the aggregate in which is ⅜ in., acts in conjunction with the precast slabs as the web of the girder and is reinforced diagonally to resist shearing stresses. The main tensile reinforcement in the girder is fourteen ½-in. bars extending the entire length of the roof just above the eaves. Reinforcement projects from the rafters of the frames and from the purlins into the screed, which is thicker than 1 in. over these members. The top face of the precast slabs is rough in order to bond with the screed.

Assembly Hall.

The frames in the assembly hall are at 5 ft. 6 in. centres and the height to the eaves, where a haunch is provided, is

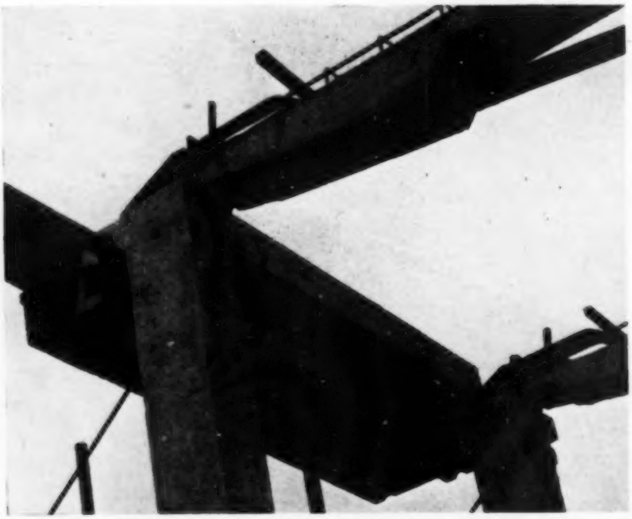


Fig. 5.—Supports of Trussed Beams.

Fig. 6.—Gymnasium Roof.

37 ft. The cross section of the posts is 18 in. by 7 in. At 12 ft. above the ground floor there is an intermediate floor (*Fig.* 2), which comprises a composite slab of precast slabs and cast-in-situ concrete, and trussed beams of 27 ft. 6 in. span carried on cantilevers monolithic with and projecting from the posts of the frames. The frames in their permanent condition are assumed to be three-hinge frames with a tie at the level of the intermediate floor, but they are designed to act, from the time of erection to the completion of the construction of the floor, as three-hinge frames with a bending moment applied at the level of the cantilevers. The roof supported by the frames has no purlins, but precast slabs, similar to those already described, span between the frames. There is no in-situ concrete over the slabs except in the recesses at the rafters. Acoustic plaster is to be applied to the underside of the slabs and the side and bottom of the rafters. The covering over the slabs is to be cedar shingles.

Each frame is in halves, each of which weighs about 4 tons. The half-frames

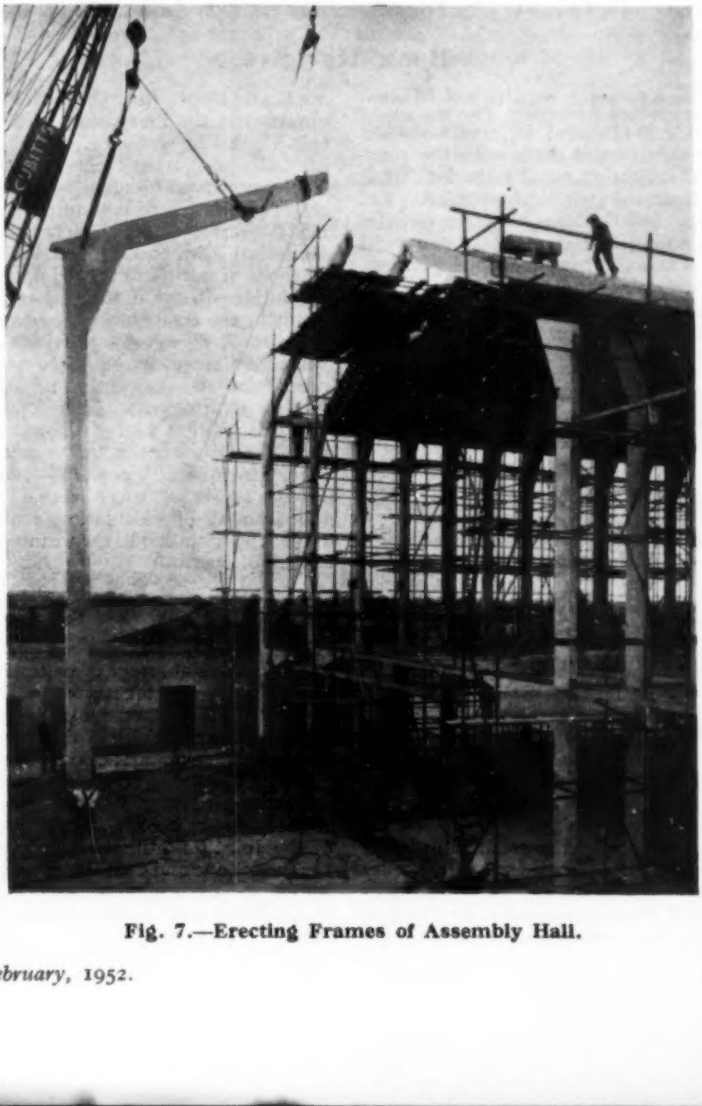


Fig. 7.—Erecting Frames of Assembly Hall.

were cast on the site in timber moulds with concrete bases, and the first operation in erection was to turn the frame so that it rested on the outside edge of the post with the rafter in the air. The next operation was to raise the half-frame into an almost vertical position by suspending it from two points on the rafter. While getting into this position, the bottom of the post dragged along the ground on a wooden skid while the rafter was raised by a 4-tons swan-neck crane with a 60-ft. boom and 20-ft. jib. The crane was self-propelled on endless track and, with the half-frame suspended clear of the ground (*Fig.* 7), travelled to the position in the building in which the frame was to be erected. The half-frame was then lowered on to its foundation from which two bolts projected and engaged in slots in a steel plate fixed to the bottom of the post.

The architects are Messrs. Easton & Partners, and the consulting engineer Mr. F. J. Samuely. The general contractors are Messrs. Gilbert-Ash, Ltd. The precast concrete members were made by Messrs. Holland & Hannen and Cubitts, Ltd., who also undertook their erection.

Book Reviews.

"The Modern Factory." By E. D. Mills. (London: The Architectural Press. 1951. Price 30s.)

THIS book is intended for architects and factory owners and deals with the planning and construction of factories. It is well illustrated with photographs and drawings, and contains chapters on sites and planning, factory estates, structural and technical considerations, stores, administration, laboratory, and welfare buildings. There are several charts and other data on lighting, thermal insulation, ventilation, and similar matters, and some details of building construction. A short chapter on structures describes some of the newer methods of factory construction in concrete and steel but does not give details of an engineering nature.

"Die Zweiseitig Gelagerte Platte." By H. Olsen and F. Reinitzhuber. Volume II. (Berlin: Wilhelm Ernst & Sohn. 1951. Price 32.00 D.M.)

THIS volume on the design of slabs supported on two edges gives data for the application of the theoretical results obtained in the first volume (reviewed in this journal for January, 1951) by means of influence areas for various loads. Tables are given for slabs having different ratios of lengths of sides from which intermediate cases can be interpolated. Six numerical examples explain the method. Methods of calculation based upon current regulations do not, in the authors' opinion, result in the economical use of materials. An example given is the provision of transverse reinforcement equal to a constant proportion of the main reinforcement. In the limiting case of a long narrow slab this requires a large amount of transverse reinforcement, whereas according to theory the quantity required is very little. It is, however, questionable whether the amount of calculation required by the authors' method is warranted by the saving of cost.

High-Tensile Steel for Prestressed Concrete.

IT is announced that the Ministry of Works will be responsible for the allocation of high-tensile steel for prestressed concrete, and a committee representing the Prestressed Concrete Development Group and the concrete products industry will act on behalf of the Ministry. The committee is now asking users for estimates of their requirements. Returns for high-tensile steel have to be made by products manufacturers where the prestressing is done in a factory, by builders when the prestressing is done on the site, and by manufacturers who supply cables to products manufacturers or builders. Details of the scheme can be obtained from the secretary of the Prestressed Concrete Development Group, Cement and Concrete Association, 52 Grosvenor Gardens, London, S.W.1, in the case of steel for in-situ work, or from the secretary of the Joint Co-ordinating Committee for the Cast Stone and Cast Concrete Products Industry, 17 Amherst Road, London, W.13, if the steel is required for products made in a factory.

A Retaining Wall with Precast Slabs.

A REINFORCED concrete retaining wall about 30 ft. high was recently constructed at Dunaskin brickworks, Ayrshire, with precast slabs placed between cast-in-situ ribs (*Fig.* 1). The wall replaces a masonry-faced wall 55 ft. high, part of which had collapsed. A shelf of whinstone, on which the old wall was founded, exists about 8 ft. below the ground in front of the wall. The back-fill is made-up ground.

The space in front of the old wall was restricted, and excavation of the back-fill would have required heavy shoring and might have endangered the stability of machinery at the top of the slope. These conditions made an ordinary retaining wall impracticable. The vertical reinforced concrete ribs are at 15 ft. 3 in. centres and are connected at the bottom by a continuous reinforced concrete toe-beam let into the rock to prevent forward movement. The heel of each rib is tied into the rock by a 9-in. diameter reinforced concrete tension dowel. The combined action of the toe-beam, bottom slab, and dowels resists the overturning effect due to the pressure of the back-fill. The base of the wall, the toe-beam, and the ribs were cast either against the face of the excavation or in brick shuttering. The shuttering of the ribs was built up in

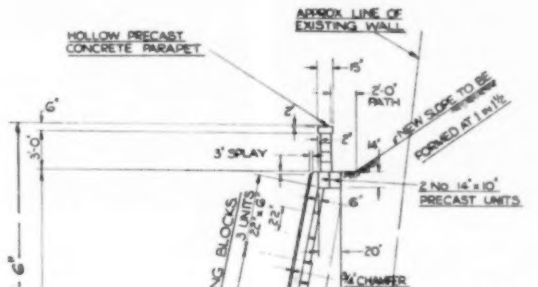
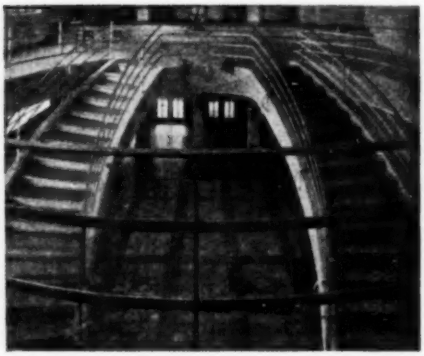


Fig. 1.

lifts. The number of precast slabs corresponding to a lift was then placed in position by a bridle suspended from a dragline with a 50-ft. jib. The wall was built in four courses of slabs at a time in the lower part and three courses in the upper part. When the slabs were in position, a timber shutter was fixed in front of them to form the forward projecting part of the rib, the reinforcement of the ribs was fixed, and concrete placed, thus securing the ends of the slabs in the ribs. The back-fill was then brought up to the level of the concrete, and a further lift commenced. No scaffolding was used.

The wall was built for the National Coal Board. The consulting engineers were Messrs. F. A. MacDonald & Partners, and the contractors Messrs. George Wimpey & Co., Ltd.

A Staircase in Germany.



The staircase shown in the illustrations has been built, to the design of Dr. Adolf Kleinlogel, at a cinematograph theatre in Karlsruhe. The space under the lower flight is solid in order to provide the weight necessary to prevent the landing and the elliptical staircase from overturning. The lower landing is 11 in. thick and is cantilevered from the top of the lower flight of steps. The top of the elliptical staircase is carried on a steel beam spanning the full width of the hall, and the bottom is supported on a reinforced concrete beam on the edge of the lower landing. The reinforced concrete stringer beams of the elliptical staircase are 13¾ in. deep by 4½ in. wide, and the reinforced concrete slab is 5½ in. thick exclusive of the steps. The sides and the upper surfaces of the staircases and landings are faced with special concrete slabs ¾ in. thick. The elliptical staircase was designed for a live load of 103 lb. per square inch. The illustrations are reproduced from the German periodical " Beton- und Stahlbetonbau."

Specifications and Quantities for Road and Bridge Works.

The Ministry of Transport has recently published two documents relating to contracts for the construction of roads and bridges. " Specifications for Road and Bridge Works " (price 5*s*. from H.M. Stationery Office) comprises all the clauses likely to be required in a specification for the construction of carriageways, cycle-tracks and footpaths of concrete or bituminous materials, concrete (excluding prestressed concrete) and steel bridges, culverts, subways, and retaining walls. Brickwork, masonry, and timber are included, as are also clearing the site, fencing, drainage, and earthworks. Drawings giving typical details of reinforcement in concrete roads are given. The other document, " Notes on the Preparation of the Bill of Quantities " (price 2*s*. from H.M. Stationery Office), deals comprehensively with the subject and gives numerous examples.

Prestressed Concrete Cantilever Bridge.

A METHOD of prestressing concrete was used for the first time in France for a bridge (*Fig.* 3) at Vaux-sur-Seine. The structure comprises two cellular double-cantilever girders, each end span being 49 ft. long, and the middle span 85 ft. The depth of the construction at mid-span of the middle span is 24 in. Each girder is prestressed by four cables arranged so that the precompression is greatest in the top over the piers, where the girders are 4 ft. 3 in. deep, and in the bottom at midspan of the middle span. The cables are anchored in the solid ends of the girders ; in one girder the anchorage is as in *Fig.* 1 and in the other as in *Fig.* 2. To allow for longitudinal deformations due to change of temperature and the application of the prestressing force, the girders are fixed to one intermediate pier by a Freyssinet hinge and supported on the other by a concrete roller-bearing. The lightly-reinforced girders were cast

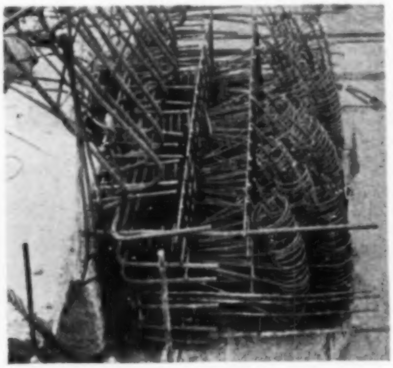


Fig. 1.

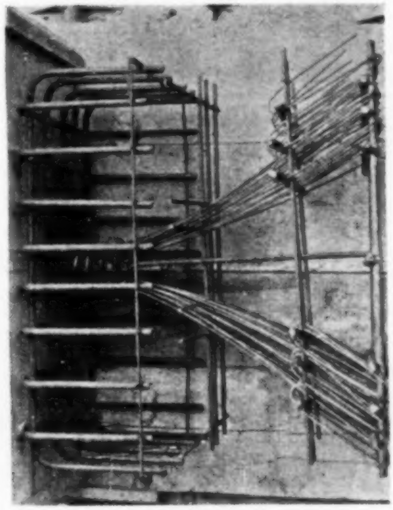


Fig. 2.

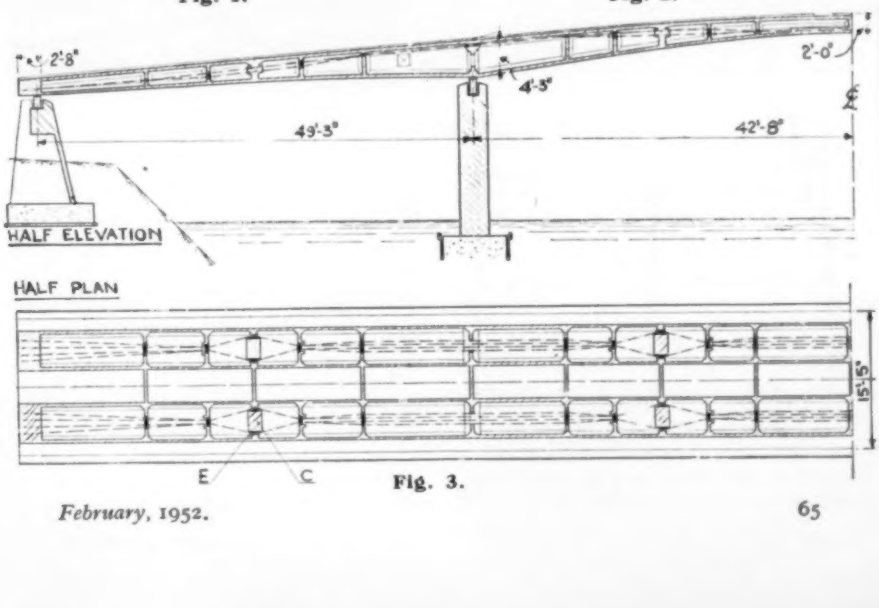


Fig. 3.

Fig. 4.

in halves on the river bank and when in position were connected at midspan by cast-in-situ concrete before being prestressed.

The girders were prestressed separately. Each cable comprises thirty-six 0·2-in. wires, which were well greased, and deviate in plan as shown in *Fig.* 3. At four places in the length of the bridge, concrete members C, which have curved steel edges E, produce the deviation. The cables were stretched by a jack (*Fig.* 4) which expanded laterally the member C, which was wedged when in the expanded condition. The method of prestressing was devised by the Société Constructions Edmond Coignet. The illustrations are from " Travaux."

Measuring the Tension in Stressed Wires.

An apparatus for measuring the tension in stretched wires has been designed, made, and tested for over two years by the " Berlin-Dahlem " Research Institute, and is described by E. Jung and A. Kretschner in " Betonstein-Zeitung " for June, 1951.

The meter (*Fig.* 1) has two hooks (1) and (2) fixed to its case (3). These hooks are fastened to the wire to be tested with the lever (4) in the position shown dotted. The lever is then turned into the position shown in full lines, thus forcing the leaf-spring (5) against a pin (9) bearing on the wire. The resulting deflection of the wire is shown on the dial (6) by the pointer (7), and is related to the force in the wire.

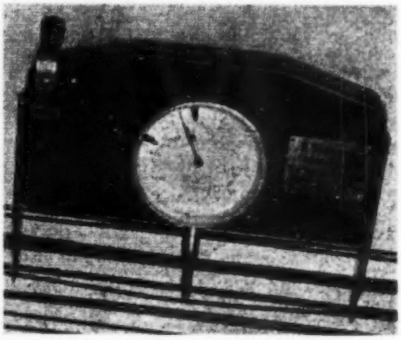


Fig. 2.—Meter in Use.

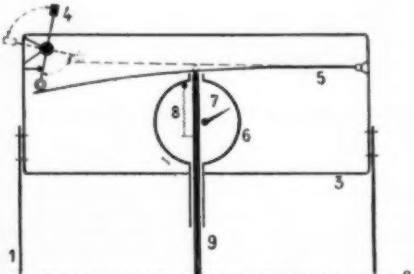


Fig. 1.—Diagram of Meter.

In prestressed concrete work, the meter may be fixed to the wire and the pull applied until the pointer shows that the required tensile force is present in the wire. Different leaf-springs may be fitted for use with wires of different diameters.

Fig. 2 shows the meter in use testing a number of wires separately. The meter is light in weight and easy to use.

A Long Concrete Culvert.

THE first of several surface-water culverts forming part of the drainage for Crawley New Town is in course of construction and is 1296 yd. long. The culvert is of rectangular cross section (*Fig.* 1), the bottom and roof being of reinforced concrete and the walls of plain concrete. For a length of 411 yd. the culvert is 5 ft. wide, for 357 yd. it is 6 ft. 6 in. wide, and for 528 yd. it is 9 ft. 3 in. wide. The height is only 4 ft. because the streams into which the culvert discharges are shallow and the land through which it passes is almost level. A similar design and method of construction are used for the three sizes of culvert, the 12-in. and 15-in. plain concrete walls being recessed into the bottom and roof as shown in *Figs.* 1 and 2. The minimum thicknesses of the bottom are 8 in., 9 in., and 11 in. in the 5-ft., 6-ft. 6-in., and 9-ft. 3-in. culverts respectively. The roof, the thickness of which is 1 in. less than the minimum thickness of the corresponding bottom, is generally just below the level of the ground and the principal reinforcement (*Fig.* 1) is the same as that in the bottom except in the 5-ft. culvert. The thickness of concrete under or over the main transverse bars is $1\frac{1}{2}$ in., except at the ends of the bars and over the main bars in the bottom slabs where it is 2 in. The longitudinal bars overlap for a length of not less than 12 in. The bottom and roof are designed as simply-supported members as the connection to the wall does not provide resistance to bending moments.

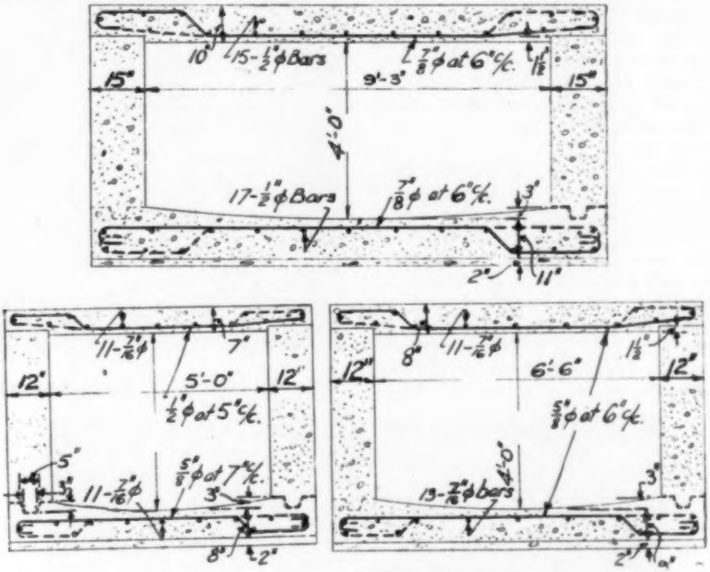


Fig. 1.—Cross Sections of Culverts.

The culverts are in clay of variable quality. The trench is excavated to a greater width than the culvert and lean concrete blinding 2 in. thick is laid in the bottom. The bottom is constructed well in advance of the walls. The proportions (by weight) of the concrete are 1 : 5 in the bottom and roof and 1 : $7\frac{1}{2}$ in the walls. The size of the gravel aggregate is from $\frac{3}{4}$ in. down in the bottom and roof and $1\frac{1}{2}$ in. down in the walls. Concrete is mixed in a $\frac{1}{2}$-cu. yd. mixer mounted on a stationary lorry. The materials are weigh-batched in a portable skip, with weighing balance attached, which discharges into the loading skip. Each batch is discharged into a hopper fixed to the tail of a small lorry which

transports the concrete to the part of the culvert required and discharges down a short inclined chute attached to the skip at right-angles to the direction of travel of the lorry. When concreting the bottom of the trench, the concrete passes from the chute into a hopper-head on a short vertical trunk extending to the point of placing.

Shuttering is provided for both sides of the walls. Steel angles are driven into the ground at 12-in. to 18-in. centres just beyond the edges of the bottom and are strutted at the top from the sides of the trench. The angles act as soldiers to retain the panels of shuttering for the outer face of the wall. Steel cross-bracing, pinned as shown in *Fig.* 2 to the panels for the inner face, maintains the inner shuttering in position, the correct thickness of the walls being obtained by timber distance-pieces. The shutters on the inner face are of No. 10-gauge sheet steel on steel angle-frames. Because of the difficulty of obtaining sufficient sheet steel, the panels on the outer face are lined with ¼-in. asbestos board manufactured by being bonded under heat and pressure with thermo-synthetic resin, and known as Pluto board. The panels of shuttering are 5 ft. by 4 ft., and the board, which does not warp in contact with the moisture, is attached to a steel

Fig. 2.—Detail of Wall Shuttering.

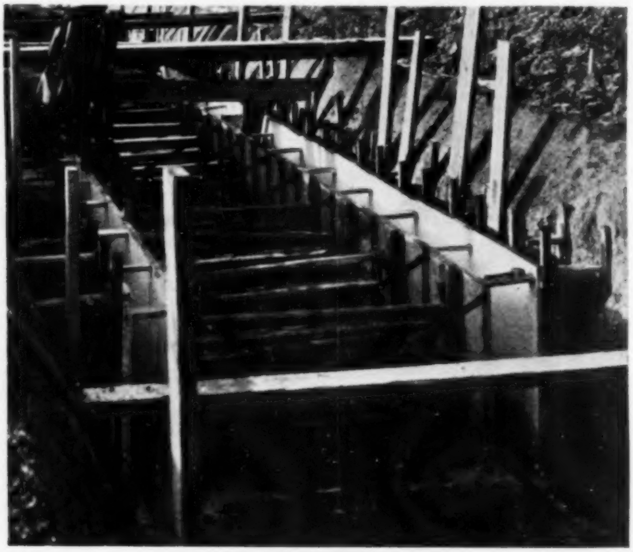


Fig. 3.—Shuttering for Walls.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frame by countersunk screws at 4-in. centres. The frame comprises 2-in. by 2-in. angles around the edges of the panel and central stiffening angles vertically and horizontally. The board can be worked with ordinary wood-working tools, and damaged panels are easily repaired. The shuttering is coated with limewash before concreting, and it is expected that the panels will be used about a hundred times. Before the shuttering of the walls is removed and the soldiers withdrawn, struts are fixed to retain the sides of the trench and support the shuttering for the top.

The culvert, which is being constructed by Messrs. Willment Bros., Ltd., was designed under the supervision of the Chief Engineer of the Crawley Development Corporation.

Air-entrained No-Fines Concrete.

In the Journal of the American Concrete Institute for June, 1951, Mr. R. C. Valore and Mr. W. C. Green describe the manufacture and tests of no-fines concrete in which air is entrained. The concrete contained rapid-hardening Portland cement, 20 per cent. to 30 per cent. of entrained air, siliceous pea gravel, and a proprietary resin or detergent air-entraining agent. It was mixed in an ordinary rotating tilting-drum mixer. The maximum air content in concrete having a compressive strength of 500 lb. per square inch at 28 days, was 25 per cent. for concrete (A) containing 310 lb., and 29 per cent. for concrete (B) containing 526 lb. of cement per cubic yard. Great difficulty was experienced in controlling the amount of air entrained.

The saturation coefficient and capillarity were much lower, the resistance to freezing and thawing generally higher, and the thermal conductivity 30 per cent. to 40 per cent. lower than for ordinary dense concrete, but the drying shrinkage was about the same. The compressive strength decreased about 100 lb. per square inch for each 1 per cent. increase in air content.

The quantity and method of adding the mixing water affected the quantity of air entrained, even if the amount of agent was constant. The density of concrete having a compressive strength of 500 lb. per square inch at 28 days was about 113 lb. per cubic foot for concrete (A), and about 105 lb. and 109 lb. per cubic foot for concrete (B) made with resin and detergent air-entraining agents respectively. The ratio of modulus of rupture to compressive strength was from 0·25 to 0·4 for compressive strengths from 300 lb. to 800 lb. per square inch, the lower ratios applying to the stronger concretes. The dynamic elastic modulus was between 1,000,000 lb. and 2,000,000 lb. per square inch, and the corresponding Poisson's ratio was about 0·18 for concrete (A) and 0·25 for concrete (B). The ratio of the secant modulus, for a strain of 0·0005 in. per inch, to dynamic elastic modulus was about 0·5 for (A) and 0·6 for (B); the secant modulus appeared to be related closely to the compressive strength and to the dynamic moduli of elasticity and rigidity.

Low values of maximum slip in bond tests and of maximum strain in the stress-strain determinations indicate that the concrete is more brittle than ordinary dense concrete. The ratio of pull-out bond strength to compressive strength was from 0·25 to 0·5, the lower ratios relating to the stronger concretes.

Research on Concrete.

THE following are abstracts from the report for 1950 of the Building Research Board, which was published recently, and describe some of the structural investigations completed or in progress.

Thin Concrete Walls.

Information on the behaviour of thin concrete walls has been obtained from models tested to destruction, the aim of the research being to reduce the quantities of materials required in load-bearing concrete and reinforced concrete walls. Previous tests on thin walls under axial loads showed that laboratory-built walls of dimensions common in practice are as strong as "short" concrete columns of equal cross-sectional area and the same age. The slenderness ratio of similar walls was increased to 50 : 1 without reduction of ultimate strength when the top and base of the walls were imperfectly restrained. In practice, imperfections of workmanship and materials combine with non-uniform load distributions to give conditions which cannot be exactly reproduced in the laboratory. The distribution of load may suggest that a wall may be considered to be loaded axially for design purposes, but this condition cannot truly exist. The manner in which these considerations are at present accounted for in the design of columns is the only guide to treatment of thin walls. It is therefore important to know the behaviour of thin walls under small eccentricities of load to allow for imperfections, in addition to the behaviour under large eccentricities which arise from the rigidity of the joints of a structural frame.

The tests were carried out on models one-half and one-third full-size. The loading conditions were probably more severe than occur in practice, since the top and base of the wall were hinged along their full length and the load was applied parallel to and on one side of the vertical axis. The initial eccentricities varied from zero to five-twelfths of the thickness of the wall. The ultimate mean compressive stress with no eccentricity was about 70 per cent. of the cube strength of the concrete, and was no less than that obtained in previous tests with the top and base of the wall imperfectly restrained. With small eccentricities resulting in failure of the concrete in compression, the strengths were in good agreement with theoretical values calculated by the elastic theory ignoring lateral deflection of the wall and assuming an ultimate compressive stress of 70 per cent. of the cube strength. The strengths were greater than the theoretical values obtained by including the effect of lateral deflection. Within the range of eccentricity in which failure in compression gives way to failure in tension, the test results were less regular, but there was no important departure from the foregoing conditions. Plastic flow of the concrete increased the ultimate strength above that given by the elastic theory. With large eccentricities resulting in failures in tension, the strengths agreed well with theoretical values assuming a limiting tensile strength a little greater than the modulus of rupture of the concrete. For example, with an eccentricity of five-twelfths of the thickness of the wall, the strength was about 12½ per cent. of the axial strength. The tensile resistance of the concrete of a plain concrete wall is a useful contribution to strength. The effects of small eccentricities and the capacity of the wall to resist tension are closely associated with the contribution to strength of the steel in a reinforced concrete wall.

Prestressed Concrete.

The research on the properties of steel wires for prestressed concrete has been continued. In the stress-strain relationships it was observed that with some samples there was an appreciable variation in behaviour between specimens cut from the same coil. A study of the variations that may occur within one length of wire and between coils from the same batch showed that the initial pre-tension might be in error by 5 per cent. or more if the wires were stretched in long lengths by an amount calculated from the stress-strain relationship obtained in tests on a short length of wire. It seems desirable therefore that the stretching force should be measured in the pre-tensioning process for long wires.

Research on the creep of wire continues. Further series of tests, which will be of longer duration, are now being started with other samples of wire to provide data for assessing the loss of prestress which may occur in practice due to creep of the steel.

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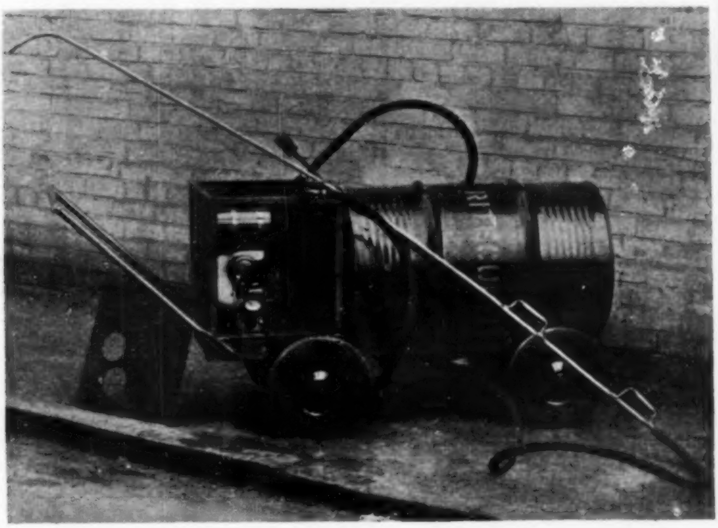

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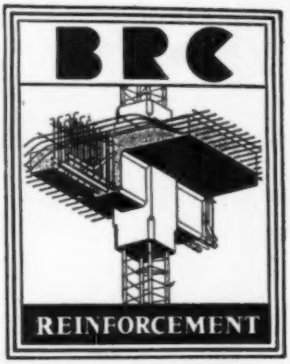
Other investigations proceeding relate to the behaviour of prestressed concrete members under static and impact loading, and the fire resistance of prestressed concrete in various types of floor construction (which show that resistance is limited by the time required to heat the steel to a temperature of about 350 deg. C., above which the steel loses strength rapidly).



"Shell" Roofs.

A method of analysis of the elastic stresses has been developed which removes many of the uncertainties inherent in some simplified methods of design of concrete shell roofs. To facilitate the application of the results, typical cases such as the stresses in a single shell with two edge-beams, twin shells, a shell resting on walls, and a cantilever shell have been worked out in detail and will be included in a later report. The theoretical work is supplemented by experimental verification of the calculated displacements.

British Standard for Metal Scaffolding.

Revised British Standard No. 1139—1951 ("Metal Scaffolding." Price 5*s.* from the British Standards Institution) specifies the material, workmanship, tests, and finish for welded close-jointed steel tubes and fittings and seamless aluminium-alloy tubes and fittings for tubular scaffolding. The standard applies also to steel trestle scaffolds (including tripods and frames) and suspended steel scaffolds.

Protection against Atomic Radiation.

CONCRETE of great density is being used in the walls to prevent radiation from a synchrocyclotron being built at the University of Chicago. The following notes on this work are from a recent number of "Engineering News-Record."

The effectiveness of a shield against the penetration of neutron radiation is roughly proportional to e (the base of the Napierian system of logarithms) raised to the power of the thickness or density. For example, a thickness of 2 ft. is 2·72 times as effective as a thickness of 1 ft. Also, for concrete, if the density is doubled, 1 ft. of the heavier material is 2·72 times as effective as 1 ft. of concrete weighing 150 lb. per cubic foot. If 4 ft. is used the effectiveness is 7·4 times as great as 1 ft., and if the thickness is 8 ft. the effectiveness is 55 times as great as 1 ft.

One shielding wall consists of precast blocks of concrete made with metal aggregates, giving a density of 300 lb. per cubic foot. (Concrete weighing as much as 370 lb. per cubic foot was made.) The wall is 12 ft. thick, $15\frac{1}{2}$ ft. high and 37 ft. long. A total of 25 blocks was required, weighing up to 100 tons each. One special block is a wedge-shaped door, mounted on wheels and weighing 67 tons. No details are given of the mixtures or type of metal aggregates used. The roof is of ordinary concrete 10 ft. thick, in the form of precast beams.

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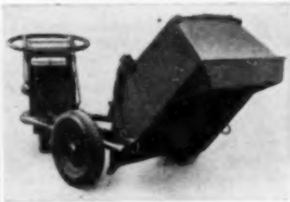

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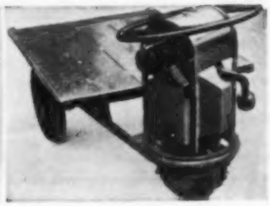

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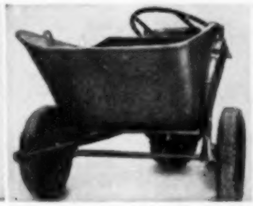

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SITUATIONS VACANT. Structural engineering designer/draughtsmen required in Designs Branch by Air Ministry Works Department. Applicants should have had several years' experience in the design and detailing of reinforced concrete or structural steelwork. The appointments will normally be in London. Salaries are in ranges up to £675 per annum with starting pay dependent upon age, qualifications and experience. Applications, stating age, qualifications, previous appointments (with dates), should be sent to AIR MINISTRY DIRECTORATE GENERAL OF WORKS, C.22 (W.9), Bush House, S.E. Wing, Strand, London, W.C.2, from which address further details may be obtained.

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SITUATION VACANT. A British firm of civil engineers and contractors operating in Malaya and Borneo requires a fully qualified and experienced engineer as principal assistant to contracts manager in supervision of contracts. Initial agreement for 3 years. Write with full particulars, both personal and professional, to Box JA/46, c/o 95 Bishopsgate, London, E.C.2.

SITUATIONS VACANT. Engineers for reinforced concrete construction required for supervision and control of contracts in India and Pakistan by J. C. Gammon, Burwood House, Caxton Street, London, S.W.1. Design knowledge valuable but not essential. Pay from £1,000 per annum, according to knowledge and experience, with normal leave passage and bonus for good work.

SITUATIONS VACANT. Simon-Carves, Ltd., require senior and junior draughtsmen for their drawing office at Cheetham Hill, Manchester, with experience in the design and detail of reinforced concrete for industrial structures. Bonus scheme and pension fund in operation. Please send full details of age, experience, and qualifications, and quote PE/724, to PERSONNEL AND TRAINING MANAGER, SIMON-CARVES, LTD., Cheadle Heath, Stockport.

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